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# Construction phases assessment for a railway tunnel

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**Abstract**

Major geotechnical issues related with tunnel construction phases are addressed. Focusing on an excavation scenario with reinforced concrete retaining walls in prevailing clayey soils, locally including sandy and silty-sandy layers, this study explores both drained and undrained conditions and considers various boundary conditions. Using the Finite Element (FE) method, a construction stage analysis of a tunnel located in the alluvial deposits of the Po Plain (northern Italy) in an urban area is carried out. This tunnel will serve the burying works of the Ferrara-Codigoro railway line, which are still underway. Based on hydrogeological conditions, the developed models take alternatively account of: (1) drained conditions with horizontal water table; (2) drained conditions with sloping water table (due to differences in water table depth between tunnel location and surrounding territories), and (3) undrained conditions with horizontal water table. Further complexity is introduced by incorporating buildings with either rigid or deformable foundations into the FE model. The fundamental role played by the boundary conditions is emphasized. Therefore, a detailed understanding of the exact sequence of construction phases and a good knowledge of the subsoil hydrogeological conditions result to be of utmost importance to capture induced settlements.



*Keywords:* construction phase; tunnel excavation; railway; settlement; drained; undrained

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## 1. Introduction

The construction stages of a tunnel constitute a challenge in geotechnical and structural engineering due to the complexity of soil-groundwater-structure interaction. This complexity is particularly relevant when these infrastructures are realized in urban areas because they could induce damage to the structures in the immediate vicinity (Giardina et al., 2015). Moreover, considering the variability of soil parameters and the possible interception of aquifers, it is necessary to explore different drained and undrained conditions, as well as boundary conditions, in order to fully grasp the soil-structure interaction. The Finite Element (FE) models demonstrate their efficiency for describing complex geotechnical problems and effectively representing the interaction between natural elements, like geological and hydrogeological conditions, and anthropogenic elements, like underground structures, e.g. Potts et al. (2001), Tezzon et al. (2015), Baraldi et al. (2018), Benvenuti and Maurillo (2019), Minkada et al. (2023).

This contribution delves into different FE models through analyses carried out by the commercial software Midas GTS NX, applied to a case study involving the construction of the tunnel of the Ferrara-Codigoro railway line. For this project, a construction stage analysis has been performed to evaluate the overall behaviour of an excavation with reinforced concrete walls in clay soils with the possible presence of sandy and peat layers, both in drained and undrained conditions and in the presence of different boundary conditions. The variation of the boundary conditions, whether hydraulic or mechanical, has allowed to explore the effect induced in terms of stress and deformation within the investigated domain. Finally, the settlements are evaluated in the presence of buildings with a rigid reinforced concrete foundation and those with a deformable foundation in crushed masonry.

## 2. Case study of Ferrara – Codigoro Railway tunnel

The case study illustrates the typical geometry of a two-track railway (Fig. 1). The excavation is supported by two retaining walls in Reinforced Concrete (RC), running parallel at a distance of 11.80 m, each with a thickness of 80 cm, reaching a depth of 18 m below the ground floor. The excavation between the walls has a width of ca. 11 m and extends underground for 8.90 m. At the base of the excavation, a reinforced concrete slab with a thickness of 0.75 m is constructed. During the excavation phase, a strut is planned to be placed at -0.60 m from the ground floor after the initial excavation of 1.0 m. This beam has an inverted T cross section, and is used as a support for the 50 cm thick slab that will be constructed upon completion of the excavation. The central distance between the beams is 7.5 m.

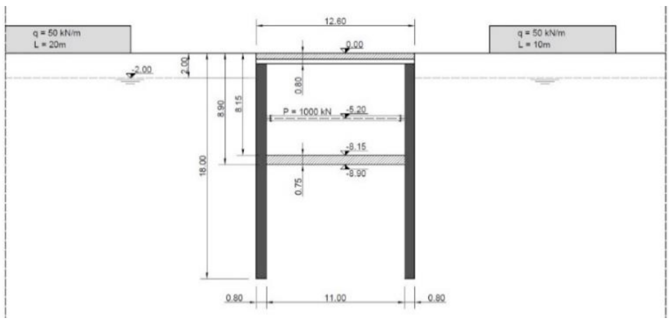



Fig. 1. Section of the excavation case study; piezometric level at 2 m b.g.l.

At a depth of 5.20 m below the ground floor, a temporary hydraulic steel strut with a preload of P = 1000 kN is provided during excavation. Along the longitudinal direction, temporary metal struts uprights with interaxis of 3.75 m are installed and will be removed at the end of the construction of the bottom and cover plates.

The prevailing soil type characterizing the excavation is clay. Although minor silty-sandy layers locally occur, for the purpose of the FE modelling a uniform clay layer has been considered down to a depth of 50 m; beneath which the presence of a bedrock is assumed. The mechanical and hydrogeological properties adopted for the soil are not specific of the construction area but are derived from the documents available at the Ferrara municipality website, i.e. geological and geotechnical reports (Fioravante and Guerra, 2008) and Rapti-Caputo and Martinelli (2008). In absence of laboratory tests to calibrate soil parameters, reference values from Ou (2006) and Kempfert (2006) are adopted.

## 3. Hardening Soil (HS) constitutive law

The appropriated constitutive law for soil modelling depends on the soil type and the applied stresses. Indeed, the stiffness that opposes loading/unloading is influenced by the load history and the level of stress reached. Finally, the selection of the constitutive law strongly relies on the analysis objectives.

A Hardening Soil (HS) constitutive law has been adopted for its ability to varying elastic modules based on the stress state (Schanz et al., 1999), essential for simulating the soil behavior during excavation. This choice is more adaptable compared to Mohr-Coulomb, which overlooks the stiffness variation as a function of the stress level and may yield inaccurate results for deformation as demonstrated by Obrzud and Truty (2018) for this type of analysis. Calibration of soil parameters involves iteratively varied parameters within GTS NX until the software faithfully reproduces the laboratory strain from tri-axial (Fig. 2) and oedometric tests through SoilTest simulator.

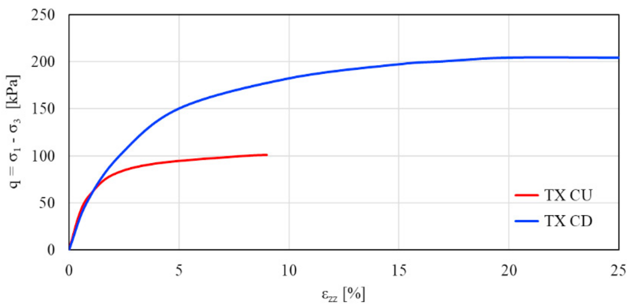



Fig. 2. Tri-axial test through SoilTest simulator: consolidated undrained test (CU) and consolidated drained test (CD).

## 4. FE Models

Various 2D FE models were made using Midas GTS NX, varying the boundary conditions at different steps. Plane strain models were employed using the hypothesis of infinite extension in the longitudinal direction.

The free-field models investigated are the following:

- model under drained conditions and horizontal water table;
- model in drained condition but with sloping horizontal water table (hydraulic gradient);
- model in undrained condition and horizontal water table.

From the various FE models for the free ground field, without structures on the sides of the excavation, the third model, undrained with a horizontal water table, was selected as a basis for further analyses:

- structures with a rigid reinforced concrete foundation with elastic modulus E = 30 GPa;
- structures with a deformable foundation of crushed masonry with elastic modulus E = 3 GPa.

The size of the calculation domain has been selected in order to minimize the edge effects: the domain extends approximately 65 m on both sides of the excavation and at depth, ensuring an overall dimension of the model domain between 6 and 10 times the depth of the excavation. The RC walls are modeled using elastic beam elements, while the struts were represented as elastic truss elements. A preload of P = 1000 kN is applied to the metal strut. Furthermore, the RC slab base is modeled as an elastic beam element. The wall-soil contact is modeled with an interface characterized by the parameters generally recommended for the clay-concrete contact. The chronological sequence of the different excavation phases is reported in Fig. 3, while the graphs of Fig. 4 represent the various meshes utilized during the excavation phases.


Fig. 3. Sequence of the excavation phases represented in months. M: mesh; L: load.

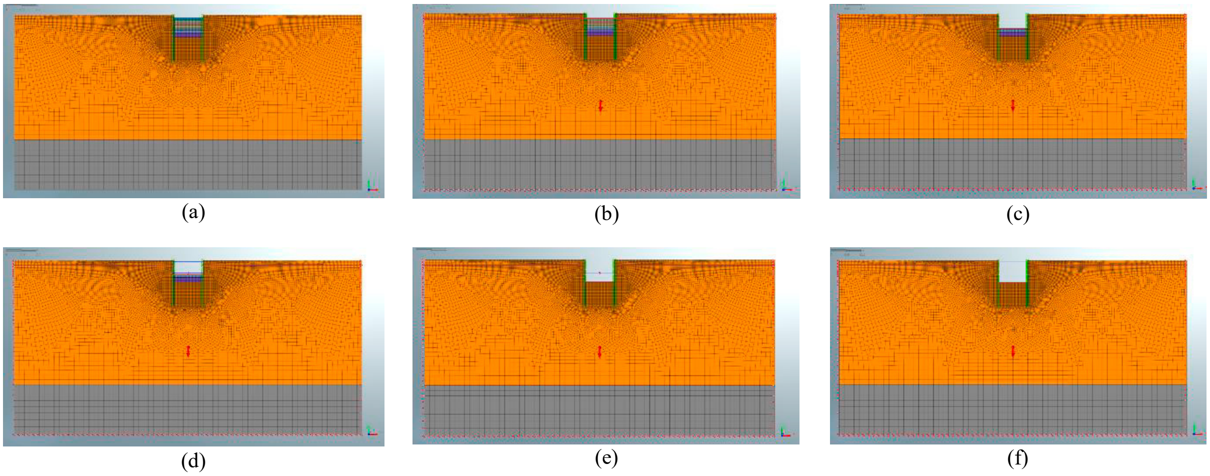



Fig. 4. Meshes used during the different excavation phases.

### *4.1. Drained models – free field*

The soil drained condition depends on its permeability, as well as the speed of load application. In this case study, the predominantly clay soil is assumed with a low permeability ($k_x = k_y = 10^{-7}$m/s), so assuming a drained condition is akin to requiring that the different excavation phases progress very slowly. This hypothesis implies that a long-time elapses between each excavation phase, allowing the filtration motions to completely dissipate the excess pore pressure. At this regard, two possible scenarios are considered:

- horizontal water table: depth -2.0 m from the ground level corresponding to the confined model (CONF);
- sloping water table: depth -2.0 m from the ground level at the left end of the modelled domain and -3.0 m to the right end. This model represents a not confined pattern (NCONF).

### *4.2. Undrained models – free field*

Due to the prevailing clay material, permeability is quite limited, and therefore it is essential to examine the behavior in undrained conditions. The short-term effect of the excavation is thus investigated. In order to carry out an undrained analysis in Midas GTS NX, it is sufficient to modify the drainage properties. The undrained Poisson's ratio is assumed to be $\nu_u$ = 0.495. This approximation of the value 0.500, typical for incompressible materials, is employed to circumvent numerical issues. The hydraulic conditions match those used for the drained model. Later, this model is referred to as UNDRAINED.

### *4.3. Drained models with soil-foundation interaction*

It is interesting to investigate the possible effect induced on the domain by the presence of existing structures adjacent to the excavation. A specially made mesh is used, with the same boundary conditions as in the UNDRAINED model. The foundations are modeled as plane strain elements with a thickness of 50 cm. The weight of the structure is modeled as a distributed load (Fig. 5). The detailed dimensions of the foundations are shown in Fig. 1.

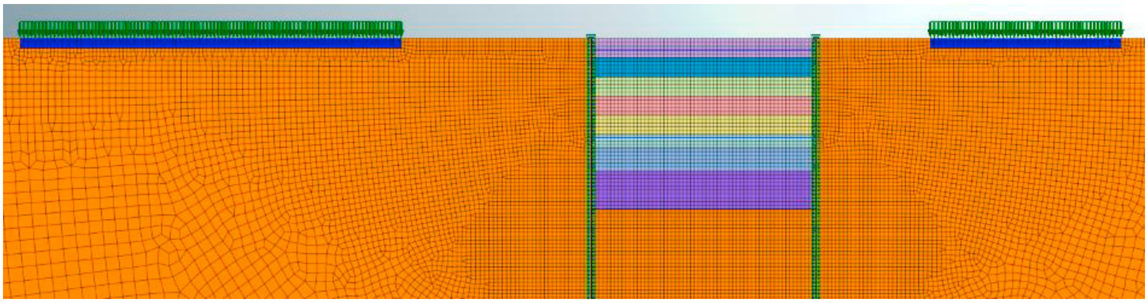


Fig. 5. Soil meshes for the FE model which includes the foundations and the structure loads associated to nearby buildings. The different colours within the excavation indicate the various phases.

When the structures adjacent to the excavation are present, a further phase is introduced compared to what has been done for the free field models. A load of 50 kPa is applied to simulate the presence of structures and the settlements produced in this phase are cancelled. Indeed, at the time the excavation begins, the superstructures have already initiated the consolidation of the soil, therefore, the displacements previously induced by the activation of the structures must be negligeable.

## 5. Results

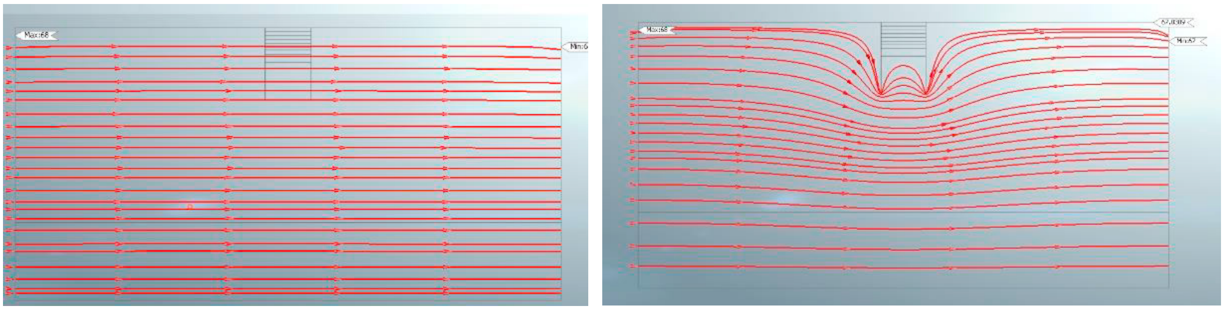
The main results obtained by the removal of the temporary metal strut are presented below. The total vertical displacements of the ground level are considered representative of the whole process.

### *5.1. Drained models*

Drained models permit to investigate the influence of hydraulic boundary conditions. In the CONF configuration, the hydraulic boundary conditions are imposed *a priori* in correspondence of all the points within the domain; with the aquifer outside the excavation fixed at -2 m from the ground floor.

In the NCONF model (Fig. 6), conditions are imposed at the right and left ends of the domain, leaving boundary conditions free at intermediate points. This modelling leaves the groundwater free to move at points between extremes. The incoming flow involves a variation of the layer level at the intermediate points of the domain during the sequence of construction phases.

The various hydraulic boundary conditions affect the displacements observed in the analysis. It is worth noting that the high hydraulic gradient, set at approximately 1%, aims to appreciate the parameter's influence on the results. Fig. 7 shows the vertical displacements recorded at the removal of the temporary strut in the CONF and NCONF models.



(a) (b)

Fig. 6. (a) Undisturbed hydrogeological conditions: flow lines in NCONF model, (b) flow lines after building the retaining walls.

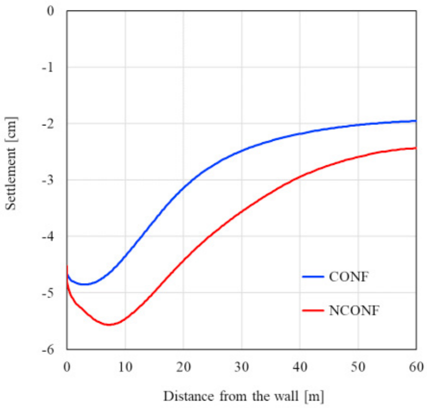



Fig. 7. Settlement profile for confined (CONF) and non-confined (NCONF) model following the metal strut removal phase.

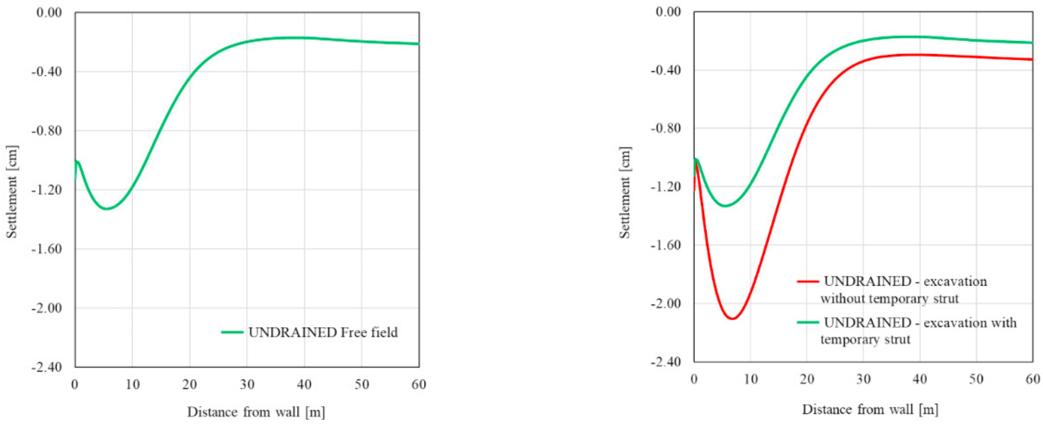
### *5.2. Undrained model*

The short-term effect of the realization of the construction is observed in the undrained models. The settlement profile at the removal of the temporary struts is shown in Fig. 8a. Notably, the maximum vertical displacement obtained from the analysis is recorded at a distance of 0.5 $H_e$, where $H_e$ the maximum depth of excavation, i.e. 8.90 m. This is in agreement with Hsieh and Ou (1998). Furthermore, it is observed that the inflection point of the curve is at a distance from the wall of 16 m, which is approximately equal to $2 \cdot H_e$, a value reported in Hsieh and Ou (1998).

Our numerical results also emphasize the crucial role in terms of containing vertical displacements that temporary struts play when properly installed, especially if compared to the case without them (Fig. 8b).

Fig. 9 depicts the color map of the vertical displacements across the calculated domain for the UNDRAINED model without the temporary metal strut.

The comparison of the results obtained with the UNDRAINED-RC and UNDRAINED-M models enables to investigate the effect of the foundation-ground interaction on the settlement profiles. As the stiffness of the foundation increases, its deformability decreases. Specifically, the concrete foundation exhibits less angular distortions than the masonry foundation. Moreover, when considering a foundation of length L = 16 m (Fig. 10b), it becomes apparent that a decrease in stiffness leads to increased deformability compared to the case with L = 10 m (Fig. 10a). All the foundations have an applied load equal to 50 kN/m$^2$.




(a) (b)

Fig. 8. Settlement profile for UNDRAINED model: (a) case with the removal of temporary metal struts in the free field, (b) case with and without temporary metal struts.

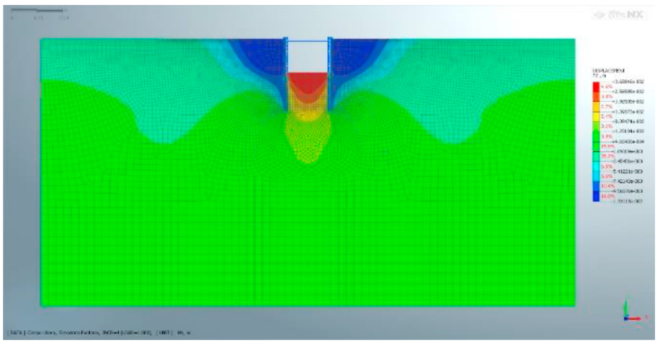
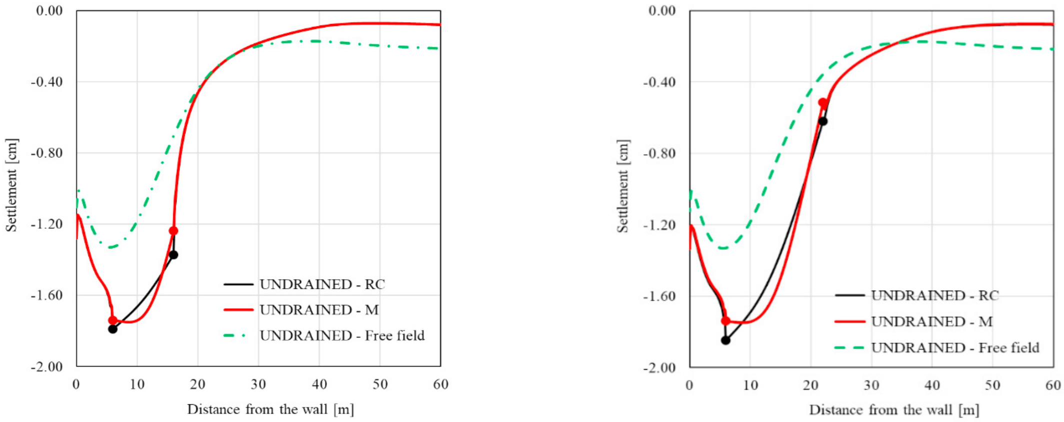


Fig. 9. Vertical displacement color map for the UNDRAINED model.




(a) (b)

Fig. 10. Settlement profile with different length of foundations: (a) L = 10 m, (b) L = 16 m.

## 4. Conclusion

The FE models show the influence of boundary conditions on total vertical displacements of the ground level. Though applied to a single excavation phase, this aspect is evident across all phases. Therefore, a detailed understanding of the construction phases, paired with a proper knowledge of the hydrogeological boundary conditions and the soil drained conditions became crucial for better understanding the potential impact on adjacent buildings during provisional works.

Indeed, the drained models reveal that different soil conditions involve different settlements of the ground. For example, the hydraulic condition with unrestricted groundwater produces a higher yield than a confined situation. Even the unconfined model allows to grasp the phenomenon of the barrier effect. Indeed, the installation of a retaining wall involves an important deviation of the groundwater filtration flow.

Free-field models in undrained conditions allow the study of short-term displacements induced by excavation. The results are in accordance with the literature, e.g. Clough and O'Rourke (1990), Hsieh and Ou (1998). Furthermore, preliminary evaluations indicate that the obtained stress resultants are consistent with those derived from consolidated calculation methods based on limit analysis.

Models including buildings close to the excavation area, say at a distance comparable to the excavation depth, show an increase in settlement if compared to the undrained free field model. Finally, the impact of excavation on foundations, i.e. differential settlements, and angular distortions, are more pronounced for deformable foundations.

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